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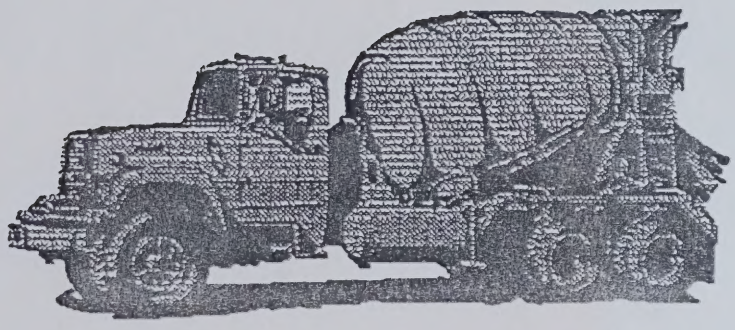

Mitigation of Potential Alkali-Silica Reactivity Using Lithium (SHRP Concrete and Structures ASR Showcase Test and Evaluation Project 34)

Study SD95-21
Interim Report

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Interim Report

Mitigation of Potential Alkali-Silica Reactivity Using Lithium(SHRP Concrete and Structures ASR Showcase Test and Evaluation Project 34)

Introduction

Since 1980 alkali-silica reactivity (ASR) has become an increasingly critical concern with regards to the durability of concrete pavements and structures in South Dakota. The first pavement to exhibit severe ASR was a 26 mile section of continuously reinforced concrete pavement on I-90 in Lyman County built in 1972. The pavement has continued to deteriorate and become a significant maintenance problem remaining serviceable only due to the presence of the reinforcing.

To combat potential problems with ASR in future construction SDDOT adopted the use of Type II, Low Alkali cement in 1983. Contemporary wisdom contended that the use of portland cement with an equivalent alkali content less than 0.6% would eliminate ASR in concrete even if an aggregate source was deleteriously reactive. Unfortunately, confidence in the assumption has been gradually eroded by the discovery of certain aggregates, mostly volcanic in origin, which undergo ASR even in the presence of a low alkali cement and by the growing awareness of deicing salts as an external source of alkali.

Since 1983 numerous pavements statewide have exhibited various degrees of ASR. In addition, Sioux quartzite, the premiere coarse aggregate used in eastern South Dakota, has proven to be slowly reactive. The impact of ASR on concrete durability and pavement life and the need to evaluate aggregate sources and develop strategies to minimize the risk of premature deterioration of PCC pavements prompted this research. Because the use of lithium salts is a recognized method of mitigating potential ASR, and because there is a need to verify the results of the Strategic Highway Research Program research on ASR, both laboratory and field testing are necessary.

Research Objectives

1) Verify the SHRP laboratory test results using lithium and fly ash with reactive and nonreactive aggregate.
2) Monitor the effectiveness of various treatments at mitigating ASR in both new and existing concrete.

Research Tasks

1) Review literature pertinent to ASR and its prevention with fly ash and lithium.

2) Conduct laboratory tests using AASHTO TP14 to confirm the results of the SHRP work using two reactive aggregate and one nonreactive aggregate with various combinations of lithium hydroxide, C and F fly ashes and a Type IP cement.
3) Construct PCC test sections incorporating various experimental combinations of cement, lithium and fly ash into concrete containing aggregate which is susceptible to ASR.
4) Treat an existing ASR-deteriorated pavement with lithium.
5) Monitor all test sections semi-annually for the first two years and annually for five years.
6) Instrument selected test sections with vibrating wire strain gauges near joints to characterize stress buildup due to expansion prior to ASR-induced cracking.
7) Obtain cores from each test section for petrographic and physical testing as needed.
8) Conduct Falling Weight Deflectometer and Impact Echo testing on all test sections.
9) Provide construction and annual evaluation reports.
10) Provide a final report and executive summary detailing a literature summary, research methodology, conclusions and recommendations.

Laboratory Tests

AASHTO TP14 was used to measure the expansion of various combinations of materials. Mortar bar tests were proportioned using the mix designs shown in Table 1. Twenty-eight mortar bar test combinations are shown in Table 2. A control sample of the non-reactive aggregate was run with each group of tests. All materials for these laboratory tests were supplied, as shown in Table 3. The chemical compositions of the cement and fly ash are shown in Table 4. Expansion values at 3, 7, 11 and 14 days are shown in Table 5.

Material Sources for Laboratory Testing

To repeat and verify the SHRP work as accurately as possible, the original material sources have been selected for the reactive and non-reactive aggregates, the cement, the fly ash, and the $LiOH{\cdot}H_2O$. It is recognized however, that due to their variability, not all of the materials may be identical to those of the SHRP study. The reactive aggregate is a mixed sand and gravel composed of rhyolite to andesite from Albuquerque, New Mexico. It reacts rapidly with low alkali cement, and SHRP measured 14 day expansions around 0.87%. The non-reactive aggregate is a quarried limestone from Rock Island, Illinois. It is non-reactive with high alkali cement, and SHRP measured 14 day expansions around 0.03%. The cement is an ASTM Type I with an alkali content of 0.2%, and is from Louisville, Kentucky. The Class C fly ash has a high lime content, and is from Rock Port, Illinois. The Class F fly ash has a low lime content, and is from Rock Port, Illinois. The chemical values and other measures for this cement and fly ash are shown in Table 4. The $LiOH{\cdot}H_2O$ is from the FMC Corporation, and their Material Safety Data Sheet is attached. The local aggregate used in the tests is an aggressively reactive sand containing volcanic materials and known to cause visible ASR distress in concrete within 5 years of construction .

Table 1: Mix Proportions for One Batch (3 bars)

Material Proportions for each Type of Material Combination					
	Admixture in Mortar Bar	Aggregate	Cement	Water	Admixture
1	None	990 gm	440 gm	220 gm	none
2	C flyash	990 gm	374 gm	220 gm	66 gm
3	F flyash	990 gm	330 gm	220 gm	110 gm
4	$LiOH{\cdot}H_2O$	990 gm	440 gm	220 gm $LiOH{\cdot}H_2O$ solution	$LiOH{\cdot}H_2O$ mixed with H_2O, see left
5	C flyash & $LiOH{\cdot}H_2O$	990 gm	374 gm	220 gm $LiOH{\cdot}H_2O$ solution	66 gm flyash (plus $LiOH{\cdot}H_2O$ mixed with H_2O, see left)
6	F flyash & $LiOH{\cdot}H_2O$	990 gm	330 gm	220 gm $LiOH{\cdot}H_2O$ solution	110 gm flyash (plus $LiOH{\cdot}H_2O$ mixed with H_2O, see left)
7	None, bars in 0.6N	990 gm	440 gm	220 gm	none
8	None, bars in 1.0N	990 gm	440 gm	220 gm	none
9	C flyash, bars in 0.6N	990 gm	374 gm	220 gm	66 gm
10	C flyash, bars in 1.0N	990 gm	374 gm	220 gm	66 gm
11	F flyash, bars in 0.6N	990 gm	330 gm	220 gm	110 gm
12	F flyash, bars in 1.0N	990 gm	330 gm	220 gm	110 gm

Evaluation of TP 14 Test Results

Reactive Aggregate

As the results in Table 5 indicate, LiOH was capable of reducing potential reactivity in the case of the New Mexico reactive aggregate in every mix where the LiOH was added to the soak solution, with or without a C or F fly ash but the LiOH added directly to the bars did not suppress expansion adequately. The failure of the LiOH in the reactive mortar bars (Mix 6) may be do to leaching of lithium ions from the bars into the solution reducing the amount of available Li^+ ions below the threshold necessary to convert sufficient ASR gel reaction product to a nonexpansive form and minimize expansion. The C fly ash (Mix 4) at 20% substitution reduced expansion slightly but definitely failed to mitigate the ASR reaction. The F fly ash (Mix 12), on the other hand, significantly influenced expansion with a 14 day expansion value of 0.108 %, but this value still exceeds both the nonreactive criteria of 0.08 % (SHRP) and 0.1 % (ASTM C1260). The combination mixes employing both lithium and fly ash in the mortar bars gave mixed results, undoubtedly complicated due to leaching. The C ash/LiOH combination (Mix 14) did not work whereas the F ash/LiOH combination (Mix 16) did. These results indicate that the

Table 2: AASHTO TP14 Test Combinations

MATERIAL TEST COMBINATIONS				
	Cement	Aggregate	Admixture	Comments
1	Type I	Control, non-reactive	none	
2	Type I	Reactive	none	
3	Type I	Local	none	
4	Type I	Reactive	C fly ash	20% cement substitution
5	Type I	Local	C fly ash	20% cement substitution
6	Type I	Reactive	$LiOH{\cdot}H_2O$	$LiOH{\cdot}H_2O$ in bars
7	Type I	Local	$LiOH{\cdot}H_2O$	$LiOH{\cdot}H_2O$ in bars
8	Type I	Reactive	none	$LiOH{\cdot}H_2O$ in 0.6N solution
9	Type I	Reactive	none	$LiOH{\cdot}H_2O$ in 1.0N
10	Type I	Local	none	$LiOH{\cdot}H_2O$ in 0.6N
11	Type I	Local	none	$LiOH{\cdot}H_2O$ in 1.0N
12	Type I	Reactive	F flyash	20% cement substitution
13	Type I	Local	F flyash	20% cement substitution
14	Type I	Reactive	C flyash & $LiOH{\cdot}H_2O$	20% & $LiOH{\cdot}H_2O$ in bars
15	Type I	Local	C flyash & $LiOH{\cdot}H_2O$	20% & $LiOH{\cdot}H_2O$ in bars
16	Type I	Reactive	F flyash & $LiOH{\cdot}H_2O$	20% & $LiOH{\cdot}H_2O$ in bars
17	Type I	Local	F flyash & $LiOH{\cdot}H_2O$	20% & $LiOH{\cdot}H_2O$ in bars
18	Type I	Reactive	C flyash, 20%	$LiOH{\cdot}H_2O$ in 0.6N
19	Type I	Reactive	C flyash, 20%	$LiOH{\cdot}H_2O$ in 1.0N
20	Type I	Local	C flyash, 20%	$LiOH{\cdot}H_2O$ in 0.6N
21	Type I	Local	C flyash, 20%	$LiOH{\cdot}H_2O$ in 1.0N
22	Type I	Reactive	F flyash, 20%	$LiOH{\cdot}H_2O$ in 0.6N
23	Type I	Reactive	F flyash, 20%	$LiOH{\cdot}H_2O$ in 1.0N
24	Type I	Local	F flyash, 20%	$LiOH{\cdot}H_2O$ in 0.6N
25	Type I	Local	F flyash, 20%	$LiOH{\cdot}H_2O$ in 1.0N
26	Type IP	Control, non-reactive	$LiOH{\cdot}H_2O$	$LiOH{\cdot}H_2O$ in bars
27	Type IP	Reactive	none	
28	Type IP	Local	none	

Table 3: Material Amounts for Laboratory Tests

MATERIAL SUMMARY FOR EACH LABORATORY				
Material	Total Batches	Provided By	Total Weights	Source
Control Aggregate	up to 3	FHWA	3 kg	Illinois
Reactive Aggregate	12	FHWA	12 kg	New Mexico
Opperman Sand	12	laboratory	12 kg	South Dakota
Cement	up to 27	FHWA	12 kg	Kentucky
C Fly Ash	8	FHWA	750 gm	Illinois
F Fly Ash	8	FHWA	1 kg	Illinois
$LiOH{\cdot}H_2O$ in bars	6	FHWA	40 gm	FMC
$LiOH{\cdot}H_2O$ in 1.0N solution	6	FHWA	425 gm	FMC
$LiOH{\cdot}H_2O$ in 0.6N solution	6	FHWA	260 gm	FMC (total 725 gm)
NaOH	up to 27	laboratory	2.7 kg	South Dakota
H_2O (distilled or deionized)	up to 27	laboratory	6 kg	South Dakota

use of an F fly ash with lithium in concrete for ASR mitigation is probably an effective and economical approach as it could reduce the quantity of relatively expensive lithium salt required for control of ASR. Although the Type IP cement (Mix 27) reduced the expansion below 0.2 % it was not capable of altering the reaction sufficiently to insure protection from ASR deterioration.

Another approach to evaluating reactivity was applied to the data for the reactive mixes based on the method of Johnston (1). The 3, 7, 11 and 14 day expansion values were fit to a polynomial curve of the form:

$$A_2 * Time + A_1 * Time^{1/2} + A_0$$

Table 4: Chemical Compositions of the Materials Used in this Study

MEASURED VALUES OF THE CEMENT AND FLY ASH			
Description	Cement	Class C fly ash*	Class F fly ash*
SiO_2	20.37	33.00	51.40
Al_2O_3	4.49	18.90	18.50
Fe_2O_3	3.32	5.97	16.10
Total Oxides)	28.18	57.87	86.00
CaO (lime)	62.20	27.00	4.49
MgO	3.88	5.28	1.06
SO_3	2.76	2.60	1.43
L.O.I.	1.18	6.0 (max)	6.0 (max)
I.R.	0.12		
Na_2O	0.41	1.98	0.84
K_2O	0.90	0.39	2.34
Alkali**	1.00	2.25	2.38
C_3S	55		
C_2S	18		
C_3A	7		
C_4AF	10		
Specific Gravity	3.15	2.70	2.42

* ASTM C 618: Class C fly ash has a 50% minimum total oxides (sum of SiO_2, Al_2O_3, and Fe_2O_3) with a 5.0% maximum of SO_3, and is produced from the burning of lignite or sub-bituminous coal. Class F fly ash has a 70% minimum total oxides, with a 5.0% maximum of SO_3, and is produced from the burning of anthracite or bituminous coal.

(While not specified in ASTM C 618, some research suggests that the lime content of fly ash has a major influence on a pozzolan's capability to control concrete expansion due to ASR.)

** Total alkalies are expressed as sodium equivalent of ($Na_2O\% + (0.658)K_2O\%$)

Table 5: AASHTO TP14 Test Results

Mix	Aggregate	Protocol	TP14 Expansion Values			
			3	7	11	14
1	Control	none	0.001	0.002	0.008	0.005
2	Reactive	none	0.189	0.361	0.471	0.502
3	Local	none	0.036	0.194	0.276	0.310
4	Reactive	20%C fly ash	0.186	0.382	0.441	0.460
5	Local	20%C fly ash	0.062	0.122	0.160	0.171
6	Reactive	$LiOH{\cdot}H_2O$-bars	0.021	0.086	0.233	0.324
7	Local	$LiOH{\cdot}H_2O$-bars	0.008	0.018	0.040	0.051
8	Reactive	$LiOH{\cdot}H_2O$ in 0.6N	0.003	0.002	0.006	0.009
9	Reactive	$LiOH{\cdot}H_2O$ in 1.0N	0.009	0.001	0.012	0.013
10	Local	$LiOH{\cdot}H_2O$ in 0.6N	0.006	0.004	0.007	0.005
11	Local	$LiOH{\cdot}H_2O$ in 1.0N	0.004	0.004	0.004	0.007
12	Reactive	20%F flyash	0.016	0.030	0.070	0.108
13	Local	20%F flyash	0.009	0.014	0.021	0.030
14	Reactive	20%C flyash & $LiOH{\cdot}H_2O$-bars	0.031	0.094	0.157	0.199
15	Local	20%C flyash & $LiOH{\cdot}H_2O$-bars	0.016	0.025	0.035	0.042
16	Reactive	20% F flyash & $LiOH{\cdot}H_2O$-bars	0.008	0.006	0.020	0.028
17	Local	20% F flyash & $LiOH{\cdot}H_2O$-bars	0.013	0.011	0.017	0.021
18	Reactive	20% C flyash $LiOH{\cdot}H_2O$ in 0.6N	0.009	0.013	0.021	0.024
19	Reactive	20% C flyash $LiOH{\cdot}H_2O$ in 1.0N	0.014	0.021	0.029	0.029
20	Local	20% C flyash $LiOH{\cdot}H_2O$ in 0.6N	0.006	0.008	0.011	0.011
21	Local	20% C flyash $LiOH{\cdot}H_2O$ in 1.0N	0.012	0.010	0.013	0.014
22	Reactive	20% F flyash $LiOH{\cdot}H_2O$ in 0.6N	0.002	0.002	0.006	0.006
23	Reactive	20% F flyash $LiOH{\cdot}H_2O$ in 1.0N	0.002	0.003	0.006	0.008
24	Local	20% F flyash $LiOH{\cdot}H_2O$ in 0.6N	0.002	0.007	0.004	0.004
25	Local	20% F flyash $LiOH{\cdot}H_2O$ in 1.0N	0.004	0.006	0.007	0.007
26	Control, non-reactive	$LiOH{\cdot}H_2O$-bars	0.005	0.004	0.006	0.008
27	Reactive	Type IP Cement	0.008	0.029	0.118	0.179
28	Local	Type IP Cement	0.005	0.007	0.011	0.014

The A_1 term of the curve fit is plotted against the A_2 term and the location of the point on a plot of a series of reactive and nonreactive aggregate test results provides insight into potential reactivity. Figure 1 shows the results for the reactive aggregate combinations. The lines for reactive and nonreactive aggregate are those developed in the original evaluation of 30 sands in South Dakota(2). Both the control nonreactive aggregate(Mix 1) and the reactive aggregate (Mix 2) plot directly onto these lines indicating that the lines are applicable for evaluating the round robin results. The only mixes which appear to be reactive based on this method are the reactive aggregate, reactive aggregate with C fly ash and the reactive aggregate, C fly ash and lithium in the mortar bars. Interestingly, the F fly ash which had an expansion value slightly above the SHRP and ASTM criteria is shown to be effective at mitigating ASR plotting solidly on the nonreactive line. The plot indicates that the various lithium treatments are even more effective than the F fly ash.

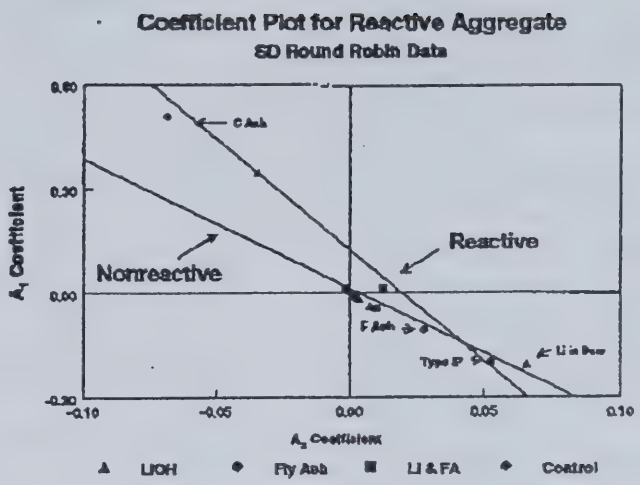

Figure 1: Coefficient Plot for Reactive Aggregate

The Type IP cement falls directly onto the reactive line on the other side of the intersection but the lithium in the mortar bars, although it is on the same side, is on the nonreactive line. Plots involving increasing amounts of fly ash exhibit the same effect with the position of the plotted point moving over to the nonreactive line to the right of the intersection just prior to moving back along the nonreactive line to the left side of the intersection. Typically, only a very slight increase in fly ash content is necessary to cause this final transition. Based on this, very little additional lithium would be needed to suppress expansion completely.

Local Aggregate

The parallel series of mixes using an aggressively reactive local sand from southeastern South Dakota illustrates how the effectiveness of any mitigation strategy is dependent on the reactive aggregate going into the concrete mix. Field performance with this sand is poor with ASR cracking evident in little more than five years even though low alkali cement was used in the concrete. Unlike the reactive aggregate from New Mexico, however, all the mixes achieved major reductions in expansion except for the C fly ash mix (Mix 5). The magnitude of leaching from the mortar bars with lithium (Mix 7), although presumably similar in the amount of the loss from the reactive mortar bars (Mix 6), did not prevent the reduction in expansion from 0.310%

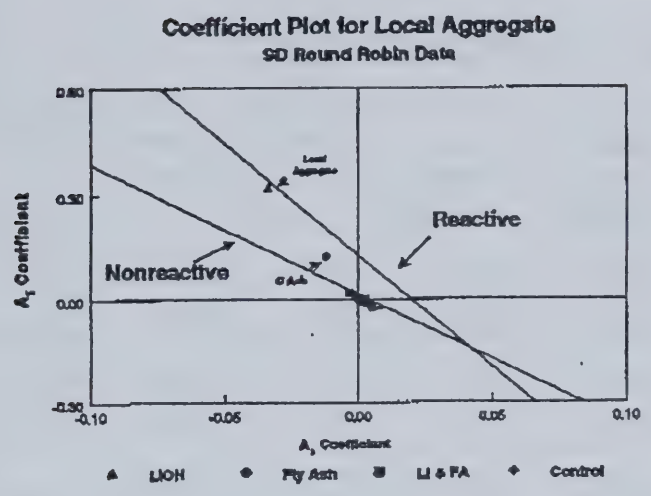

Figure 2: Coefficient Plot for Local Aggregate

to 0.051%. The implication is that the amount of lithium required for mitigation varies with the aggregate used as well as the amount of alkalies in the cement. The SHRP recommendations for lithium addition point this out but the inadequacy of using % expansion as the only criterion for different lithium dosages is indicated clearly by the results of Mix 7 (Li in bars) from Pennsylvania as compared to South Dakota. The Pennsylvania local aggregate used in the Round Robin testing had a 14 day expansion of 0.336%, not very different than the 0.310% for the South Dakota sand. Unlike the SD sand, however, the reduction in expansion with lithium in the bars was woefully inadequate with the expansion still at 0.239% versus 0.051% for the SD sand. This difference in reactivity is born out by the results using a Type IP cement. The cement worked fine with the SD sand cutting the expansion down to 0.014% whereas it could only reduce the PA aggregate to 0.160%.

Evaluation of the results with the local aggregate using a coefficient plot as shown in Figure 2 indicates only two mixes which are reactive. The first is the local sand with no admixture and the second is the sand with 20% C fly ash. All other mixes plot onto the nonreactive line.

Type I(PM) Test Results

An additional series of TP-14 mortar bar expansion tests was conducted using the same reactive aggregate from New Mexico and local South Dakota aggregate which were tested in the Round Robin. The purpose of these additional tests was to evaluate the effectiveness of a new Type I(PM) cement resulting from an earlier research project SD91-11 *Development of a Type I(PM) Cement* (3). The study comprised a search for suitable pozzolan candidates for intergrinding with normal portland cement capable of suppressing Alkali-Silica Reactivity. One particular material, a silicified volcanic tuff demonstrated excellent control of expansion in the TP-14 mortar bar test and preliminary test results on cement and concrete properties of samples made using this cement at varied addition levels were quite favorable even though the pozzolan was not calcined. All cement samples from this prior research were manufactured in the laboratory which did not necessarily reflect real production properties. The samples tested in this series were actual production samples interground at pozzolan addition rates of 5,10 and 15% by Dacotah Cement in Rapid City, South Dakota. The results of these tests are shown in Table 6.

Table 6: Type I(PM) TP-14 Test Results

Time	Local Type II	Local Type V	Local Type II Fly Ash	Local TypeV Fly Ash	Local Type IPM(5)	Local Type IPM(10)	Local Type IPM(15)
3	0.021	0.021	0.024	0.022	0.029	0.02	0.008
7	0.165	0.161	0.078	0.058	0.15	0.069	0.021
11	0.251	0.242	0.116	0.098	0.211	0.116	0.049
14	0.293	0.28	0.143	0.122	0.246	0.147	0.083
R^2	0.9999	0.99995	0.99977	0.99876	0.99963	0.9998	0.9973
Time	NM Type II	NM Type V	NM Type II Fly Ash	NM Type V Fly Ash	NM Type IPM(5)	NM Type IPM(10)	NM Type IPM(15)
3	0.128	0.051	0.092	0.047	0.126	0.096	0.052
7	0.389	0.173	0.331	0.17	0.308	0.218	0.149
11	0.525	0.253	0.456	0.248	0.411	0.292	0.227
14	0.57	0.506	0.296	0.282	0.452	0.333	0.262
R^2	0.99968	0.99995	0.99993	0.99958	0.99975	1.0000	0.9984

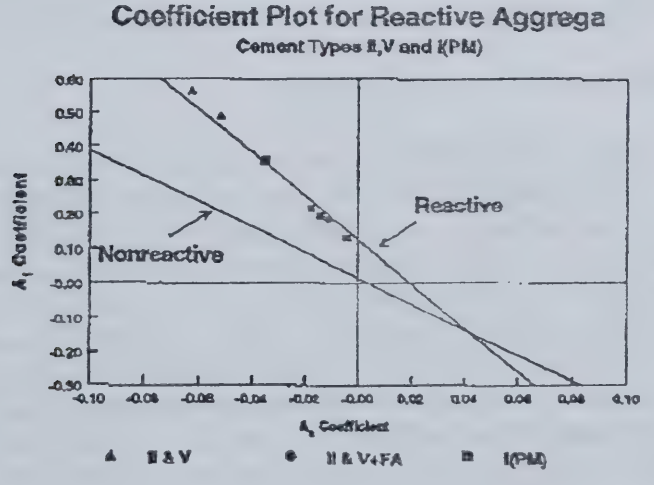

Figure 3: Coefficient Plot for Reactive Aggregate

The expansion results indicate that neither the Type I(PM) nor the fly ash (a locally available Modified F Ash) were effective at mitigating reactivity with the New Mexico aggregate but the results for the local aggregate are not entirely clear. Coefficient plots done on these results are shown in Figure 3 and Figure 4. All points for the reactive aggregate are on or near the reactive line. The points in Figure 4, on the other hand,

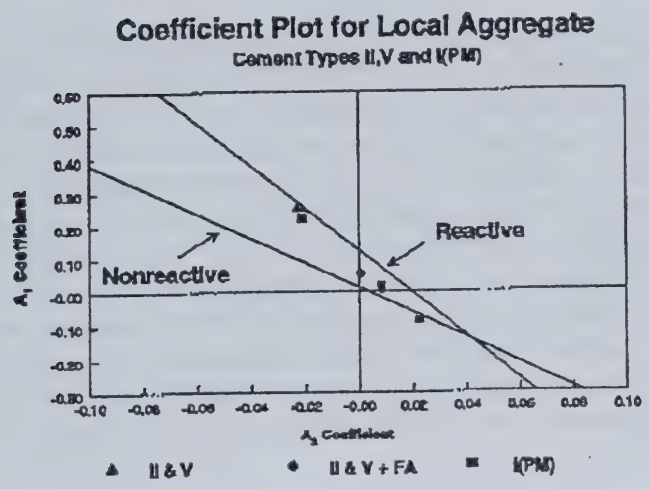

Figure 4: Coefficient Plot for Local Aggregate

indicate that the Type I(PM) cement at 15% intergrind was the only combination tested capable of stifling the reactivity of the local aggregate. Although the F fly ash and lower addition rate Type I(PM) cements plot in between the reactive and nonreactive lines, this region is associated with slow, late expanding reactive aggregates which still cause deterioration and these combinations cannot be considered entirely effective. They may slow down the rate of reactivity but deterioration will probably still occur. Another interesting aspect of these test results is the difference between the Type II (technically a Type I-II) cement and the Type V cement. SDDOT currently specifies the use of a Type V low alkali cement in conjunction with a Modified F fly ash for concrete paving projects where an aggressively reactive (% expansion > 0.25) aggregate is used. The reason for changing to a Type V is based on the assumption that the slightly different cement chemistry of a Type V should help to slow down the rate of ASR compared to a Type I or Type II. Both the expansion data and the coefficient plot seem to bear this out. There is a slight reduction in expansion using the Type V cement with the local aggregate and a significant reduction with the reactive aggregate. The Type V cement used in these tests came from the same manufacturer and control tests using Ottawa sand (to insure the cements used were not contributing to the expansion) gave zero expansion. The R^2 values were included to show a typical range of values obtained when the curve fitting procedure is applied to TP-14 test results.

ASR Expert Task Group Field Trip

A team composed of members of the ASR Expert Task Group (ETG), and representatives from each participating laboratory visited two experimental sites; one in Albuquerque, New Mexico, and the other in Winnemucca, Nevada. At Albuquerque, New Mexico, new concrete sections were constructed with several admixture treatments to test their effectiveness in reducing ASR expansion in new concrete. Fly ash and $LiOH{\cdot}H_2O$ were included in the various treatments. At Winnemucca, Nevada, existing concrete was treated with several materials to test their effectiveness in limiting ongoing ASR expansion. $LiOH{\cdot}H_2O$ was included in the various treatments. The team visited these sites to rate and rank the effectiveness of the mitigation measures. Based on the results of this field trip the team decided there was definitely a need to establish more test sections involving the use of lit[illegible] in both new and existing concrete and F fly ash in new concrete.

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Field Tests-New Concrete Construction

The project selected for construction of new concrete test section was IM90-5(80)262 on Interstate 90 near Chamberlain, SD. This project was ideal for installation of these test sections as both the fine and coarse aggregate used in the concrete pavement are reactive. The fine aggregate is the aggressively reactive Opperman-Herrick sand which was the local aggregate tested in the laboratory phase of the research and the coarse aggregate is Sioux Quartzite which was identified as slow, late expanding alkali reactive by the earlier SHRP research and was one of the aggregate tested in C-343. The project is a 8.3 mile replacement of an existing asphalt concrete pavement with continuously reinforced concrete pavement (CRCP) and jointed plain concrete pavement (JCP). All test sections where deterioration could occur were placed in the JCP sections to facilitate maintenance and possible replacement. A low alkali Type V cement/Modified F fly ash was used in all other concrete to reduce the risk of potential ASR. To facilitate construction the same low alkali Type V cement was used in all test sections. Test sections included:

1) Type V LA cement
2) Type V LA cement + LifetimeDS™ Admixture (two dosages)
3) Type V LA cement + LifetimeDS™ Admixture + Modified F fly ash
4) Type V LA cement + Lifetime™ Admixture (three dosages)
5) Type V LA cement + LifetimeN™ Admixture (four dosages)
6) Type V LA cement + LifetimeN™ Admixture (two dosages) + Modified F fly ash
7) Type V LA cement + Modified F fly ash
8) Type I(PM) cement

Concrete Mix Designs Using Lifetime Admixtures

Because the above combinations involved a total of fourteen different test sections a series of mix designs were run on the maximum dosage of each admixture. Mix designs were also run on each admixture plus fly ash, where applicable. The results are listed in Table 7 and indicate the only effect of the various lithium-based admixtures was a slight to moderate reduction in compressive strength. Prior work with these admixtures involving mixes with Type I cements showed that whether there was a slight increase or decrease in compressive strength depended on the cement used and its alkali level (4). The effect on strength seems more pronounced with the Type V cement and this, undoubtedly, is directly related to the considerable difference in cement chemistry between a Type I and a Type V cement. Notably, the original Lifetime™ admixture significantly reduced the compressive strength whereas the non OH^- containing LifetimeN™ had a minimal effect at maximum dosage.

LifetimeDS™ Admixture caused some major concerns in the mix design stage due to a premature stiffening effect which gave a totally unworkable mix with no entrained air. Similar results had been obtained with various admixtures at high dosages in a previous research project SD 92-07 *Evaluation of the Performance of High-Range Water-Reducers and Set Retarders in Typical SDDOT Concrete Mixes* (5). The solution to the problem was to wait five minutes before adding the admixture which seemed to eliminate most of the problem. The actual mix design for LifetimeDS™ Admixture was done using a five minute delayed addition and this yielded a

Table 7: Concrete Mix Designs Using Lifetime Admixtures

Ingredient	Control	Lifetime	LifetimeN	LifetimeDS	LifetimeN +FA	LifetimeDS +FA
Cement (lbs)	600	600	600	600	510	510
Flyash (lbs)	—	—	—	—	112.5	112.5
Li Admixture (lbs)	—	4.5	4.5	48	1.8	24
W/(C+F)	0.405	0.399	0.391	0.434	0.368	0.407
Water (lbs)	243	239.4	234.7	260.3	229.2	253.2
Sand	1249	1249	1249	1230	1211	1211
Rock	1740	1740	1740	1714	1759	1759
A.E.A (fl oz)	9.6	9.9	9.9	14.0	12.7	13.3
Add Water (lbs)	—	-3.57	-8.26	+12.4	-11.6	+12.4
Slump (in)	2¼	2¼	2½	2½	2½	2½
Unit Weight (lbs/ft^3)	146.8	144.4	145.4	144.4	144.6	146.2
Air Content (%)	5.3	5.6	5.3	5.4	5.6	5.0
Compressive Strength						
3 days	3337	2963	3390	2743	2700	2717
7 days	4263	3923	4140	3460	3430	3330
28 days	5807	5057	5673	4923	4950	4863

concrete with normal workability. Since using a nominal five minute waiting period in a central plant operation would be extremely difficult, a decision was made to conduct full size test batches at the central plant prior to actual construction.

Field Trials of Concrete Mixes

Lithium Based Admixtures

A trial mix of 7 yd^3 concrete made with the addition of 48 lbs of LifetimeDS™ Admixture per yd^3 was produced out at the contractor's central plant on the afternoon of Sunday, June 24, 1996. Since this particular admixture was a fine solid and no equipment existed for adding it to the mix a sheet metal chute was rigged by the contractor to introduce the powder onto the sand belt just prior to the sand entering the mixing chamber. This seemed to work well with very little material loss due to dusting. Since a five minute delay was impractical with this material and could have adversely affected paving operations the only alternative was to increase mix time and hope the concrete would mix out of the premature stiffening in a manner analogous to eliminating a false set problem. Mixing was begun using the mix design parameters in Table 7 and the amperage demand for the system was closely monitored as the mixing progressed. The plant operator noted that the current requirements were significantly higher than the normal job mix he had been producing during the last week and stayed that way after the first minute of mixing. Mixing was continued and after 1 minute 30 seconds the amperage demand began to drop. The Lifetime DS™ mix was discharged after 2 minutes and slump, concrete temperature and air content measured. The slump was 1¾", temperature was 79°F and the air content was 7.3%. Workability was somewhat stiff but no problem. The batch was driven around the plant in an agitated alligator dump truck for 28 minutes to simulate hauling and waiting before actual delivery and slump, temperature and air were again measured. They were 2", 82°F and 5.8%, respectively. Workability of the mix improved compared to the sample taken immediately. These results were somewhat surprising, to say the least, but positive enough to include the Lifetime DS™ mixes in the test sections with an increase in nominal mixing time from one minute to two minutes.

A second test batch was produced incorporating Lifetime DS™ at 24 lbs/yd^3 and 112.5 lbs fly ash/yd^3 to determine if increased mixing time would also minimize problems with this mix. The results were very similar to the first mix with air and slump at 7.3% and 2⅛" after mixing and at 5.2% and 2" twenty seven minutes later. This mix was also included in the test protocol.

Construction of Lithium Test Sections

Actual construction of test sections was begun early on the morning of June 26, 1996 due to a rain delay. A pump for introducing solutions of the lithium admixtures into the mix was provided by FMC Lithium Division, manufacturers of the Lifetime admixtures. The pump was capable of transporting about ⅓ gallon/second of liquid admixture to the top of the plant 35 feet in the air from a 50 gallon drum on the ground below some 50 feet from the plant. No equipment problems were encountered with the introduction of the various admixtures into the mixes but the crew balked at working with the caustic Lifetime™ LiOH admixture based on the information provided in the MSDS for the product. They had no qualms about the other admixtures although goggles and gloves were worn when working with all liquids with masks

added when introducing the solid admixture onto the sand belt.
Overall, the mixes went extremely well throughout the day even though numerous adjustments in water, air entraining admixture and aggregate weights had to be made. The rain the day before had soaked both fine and coarse aggregate stockpiles so that the free moisture changed throughout the day as the piles dried. The free moisture in the first batch at approximately 7:30 AM was running about 6.1 lbs/yd^3 but by the time the last experimental batch was produced around 2 PM it was down to 2.7 lbs/yd^3. The air temperature during experimental production went from 72°F in the morning to 96°F by early afternoon. Results from air content, slump, unit weight, air and concrete temperature, compressive strength and static modulus testing are shown in Table 9. The 60 day compressive strengths were based on cores obtained for static modulus testing. The static moduli values will be used to monitor the effects of ASR on concrete properties over the next four years.

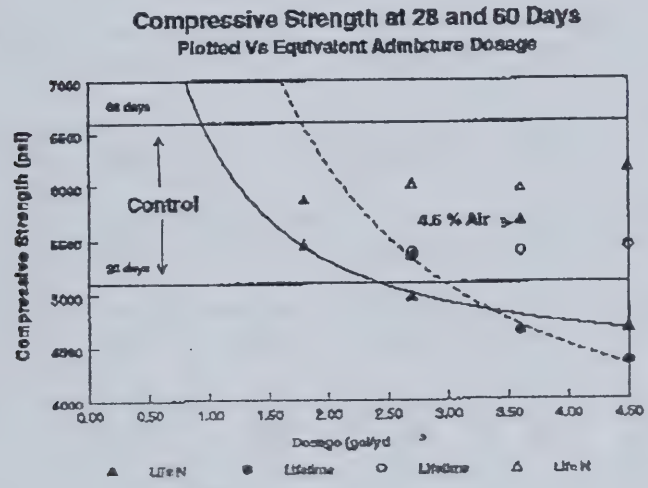

Figure 5: Compressive Strength Results

Figure 5 shows the results of compressive strength tests at 28 and 60 days. The 28 day strengths appear to be inversely related to the dosage of lithium admixture used with the strength falling off at higher dosages. The anomalously higher strength exhibited for the LifetimeN™ at an equivalent dosage of 3.6 lbs/yd^3 is readily explained by the low air content of 4.6% which was significantly lower than the other LifetimeN mixes. This seems to be a transient effect as the 60 day compressive strengths from cores show little effect due to dosage for both Lifetime™ and LifetimeN™ admixtures. Concrete mixes with both admixtures exhibited a reduction in 28 day compressive strength at higher dosages compared to the control mix with the maximum dosages resulting in a reduction of approximately 8% and 15% for the LifetimeN™ and Lifetime™, respectively. Corresponding reductions at 60 days averaged 9% and 18% with the Lifetime suppressing the ultimate strength significantly. The LifetimeDS™ mixes gave split results at 28 days with the lower dosage 9% below the control and the higher dosage 9% above the control. The 60 day core strengths, on the other hand, yielded a pattern consistent with the longterm effect of the original Lifetime™ or LiOH admixture with reductions in strength of 19% and 15% at the lower and higher dosage rates, respectively. All mixes met the specified 4000 psi strength requirement at 28 days.

Table 8: Concrete Mix Parameters for Test Sections

Mix	Cement (lbs)	Fly Ash (lbs)	Water (gal)	Coarse Aggregate (lbs)	Fine Aggregate (lbs)	Lithium Admixture (gal)	A.E.A (fl oz)
1	600	—	23.5	1751	1292	Lifetime 2.0	13.3
2	600	—	23.5	1749	1290	Lifetime 2.6	13.3
3	600	—	26.0	1738	1275	Lifetime 3.2	14.3
4	600	—	25.3	1752	1285	LifetimeN 1.0	16.1
5	600	—	25.3	1751	1284	LifetimeN 1.5	17.6
6	600	—	25.3	1749	1283	LifetimeN 2.0	17.6
7	600	—	23.3	1757	1290	LifetimeN 2.5	17.6
8	510	112	22.8	1778	1252	LifetimeN 0.5	20
9	510	112	22.8	1776	1251	LifetimeN 1.0	20.7
10	600	—	25.4	1702	1250	LifetimeDS 24 lbs	17.1
11	600	—	29.0	1649	1194	LifetimeDS 48 lbs	18.6
12	510	112	28.2	1701	1182	LifetimeDS 24 lbs	18.6
13	600	—	26.8	1749	1266	—	18.6
14	510	112	24.3	1773	1241	—	18.6

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Table 9: Results of Concrete Tests

Section	Admixture	Sample Location	Plant Air (%)	Paver Air (%)	Slump (in)	Concrete Temp.	Air Temp.	Unit Weight (lbs/ft³)	AEA (fl oz)	Water (gal)	Lithium Admix (gal)	Compressive Strength (psi)			Static Modul E (x 10^6) psi
												7 days	28 days	60 days	
1	Lifetime	51+75	7.3	6.7	1¾	78	72	144.1	13.3	23.5	2	4280	5342	5380	5.11
2	Lifetime	50+75	6.4	8	2	78	72	146.8	13.3	23.5	2.6	3838	4652	5390	5.2
3	Lifetime	49+75	7.6	7.6	1½	77	72	143.6	14.3	26.0	3.2	3732	4360	5430	4.91
4	Lifetime N	48+75	5.8	7	1½	79	81	144.2	16.1	25.8	1	4519	5448	5870	5.09
5	Lifetime N	47+75	7.2	6.8	1¾	79	82	143.8	17.6	25.3	1.5	4103	4979	6010	5.07
6	Lifetime N	46+75	7.4	4.6	1½	80	85	144.8	17.6	25.3	2	4705	5678	5980	5.12
7	Lifetime N	46+40	9.4	8	1¼	81	85	143.6	17.6	23.3	2.6	4190	4679	6160	5.27
8	Lifetime N + ash	45+50	6.6	5.2	1¾	81	89	145	20	22.8	0.5	4280	5262	5530	5.22
9	Lifetime N + ash	44+50	7	5.6	2¼	84	92	143.8	20.7	22.8	1	3830	5041	5860	5.04
10	Lifetime DS	43+90	7.8	7	2¼	83	92	144.4	17.1	25.4	24 lbs	3935	4643	5340	5.01
11	Lifetime DS	43+00	6.7	5.4	2½	80	93	145.2	18.6	29.0	48 lbs	4457	5572	5630	4.76
12	Lifetime DS + ash	42+30	7.6	5.6	2	80	96	143.2	18.6	28.2	24 lbs	4245	5147	6310	5.48
13	Control	41+95	6.6	5.4	1¾	84	96	144.3	18.6	26.8	—	4386	5103	6600	5.32
14	Normal Mix-Before	52+40	—	5.6	1¾	76	74	—	18.6	24.3	—	—	—	5740	4.98

Discussion

Though the use of lithium admixtures with the Type V cement used in constructing these test sections is atypical of normal applications where a Type I or Type II portland cement would be employed instead, there are still several pertinent items which these test mixes certainly clarified. All point to the advantages inherent in shifting away from the original LiOH (Lifetime™) admixture recommended in the SHRP research to a noncaustic lithium salt such as LifetimeN. Amongst these are:

1) The use of an admixture which does not contain or generate OH^- as a primary anion eliminates handling and safety problems associated with a high pH liquid rated as a hazardous material.

2) Avoiding the introduction of these same extraneous OH^- ions into a concrete incorporating reactive aggregates seems a prudent measure. Recent experimental evidence indicates that, unlike LiOH, Li_2CO_3 and LiF, LifetimeN™ does not generate a significant amount of OH^- ions in fresh concrete (6).

3) LiOH reduced the ultimate compressive strength of the concrete by almost twice as much as LifetimeN™ at higher dosages compared to the control mix as was born out in laboratory mixes where the difference was even more pronounced.

4) LifetimeN™ acts as a water reducer, especially at higher dosages, and this benefit can be used to partially offset any reduction in strength.

Construction of test sections with these various lithium admixtures involved minimal problems and caused no delays in paving operations. Only one air test and three slump tests were out of specification over the course of the pour and these problems were more related to the complexity of changing concrete mixes frequently than to any inherent problems with the use of these admixtures. A reduction in compressive strength, although unfortunate, is not a major barrier to using these admixtures as all strengths met specifications.

Type I(PM) Test Section

An additional test section was constructed on this same project two months later using the Type I(PM) cement developed from research project SD91-11 *Development of a Type I(PM) Cement*. The three plant produced blended cements utilized in the TP-14 testing were mixed by recirculation to give a final composite of approximately 12% addition. This mixture was used to construct a test section on the same project which incorporated the lithium admixture test sections but was located some distance west of the other test sections. The contractor decided to produce the concrete for this section at a local redi-mix plant and pave the relatively short test section after all other paving was completed. The lithium test sections had been slipformed but the contractor opted to pave the Type I(PM) section using a bridge deck paving machine on rails.

Construction was begun early on the morning of August 30, 1996. The Type I(PM) test section

was placed from Station --- to Station---. Conditions remained ideal throughout the pour with air temperatures between 60 and 70°F. Unlike the earlier slipform paving the maximum allowable slump was 3". The initial slump and air at the Redi-mix plant were 3¼" and 7.0%, respectively. A concrete sample was taken from the fifth load delivered to the test site and had a slump of 2¾" and an air content of 5.0%. Concrete placement and finishing went smoothly with no problems reported even though much of the finishing was done by hand. Table 10 gives results for compressive and flexural strength tests at 7, 28 and 90 days. Although the strength results were somewhat disappointing and lower than expected this can be explained somewhat by the increase in allowed slump coupled with a slight increase in water demand due to the pozzolan. A C109 mortar bar test of compressive strength from a grab sample of the Type I(PM) cement used on the job gave 4100 psi at 7 days. Further testing will be conducted on cores.

Table 10: Strength Results for Type I(PM) Test Section

Age (days)	Compressive Strength (psi)	Flexural Strength (psi)
7	3594	425
28	4032	525
90	5399	607

Field Tests-Treat Existing Concrete

A series of test sections employing lithium solutions (Renew™ and RenewA™) were established on in the westbound lane on sections of US 14 near Wolsey (MRM 327.43). This 7.5" jointed plain concrete pavement has 15 foot wide lanes and was built in 1981. Portions exhibit severe ASR cracking where a high alkali cement was used (Na equivalent 1.25%). A section of about 1000 ft was built using a lower alkali cement and exhibits a much smaller incidence of ASR cracking. Petrographic analysis reveals the coarse aggregate, Sioux quartzite, has undergone ASR to some extent. The fine aggregate used on this project, a sand from Northwestern Iowa, contains quantities of reactive chert and significant distress appears related to deterioration of this chert and another constituent of this sand-dolomite. Numerous particles of reacted dolomite altered to $CaCO_3$ (calcite) and $Mg(OH)_2$ (brucite) are found in the cement matrix(8).

The purpose behind applying lithium solutions to this pavement is to determine whether postreatment of pavements susceptible to ASR-induced deterioration is a viable alternative capable of slowing down the rate of deterioration and extending the life of the concrete. Both solutions used in the trial are neutral lithium salts and lithium hydroxide was not applied as it was felt the addition of OH^- ions to the concrete could possibly exacerbate the ASR. Cores were taken before and after treatment and static modulus testing was performed using ASTM C 469 *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression.* In addition dust samples were obtained to monitor the penetration of lithium ions into the concrete.

The application of Renew™ and RenewA™ solutions occurred on October 13, 1995. A tank truck equipped with ground speed control was used to apply the solutions to a series of six test sections each 500 feet long. Different dosage rates were achieved by repeated spray applications with each test section adjoining. A 500 foot untreated section was left between the Renew™ and

Table 11: Lithium Test Sections US14/281 MRM 327.43 WB

Test Section	Begin Station	End Station	Gallons (30% Solutions)	Impact-Echo Velocity	Static Modulus	
					Before	After
Renew-SL 5%	55+00	50+00	30.03	3986	2.57 3.59	– 3.45
Renew-SL 10%	50+00	45+00	58.93	4012	3.05 2.70	3.49 2.77
Renew-SL 15%	45+00	40+00	81.83	4046	3.09	3.25
RenewA-SL 5%	35+00	30+00	27.62	3989	2.74 3.22	2.92 3.54
RenewA-SL 10%	30+00	25+00	53.62	4071.5	2.90 2.91	3.00 2.90
RenewA-SL 15%	25+00	20+00	78.02	4015	2.58 3.79	2.81 3.33
Control (Low Alkali)	164+05	–	–		3.03	3.09
Control (High Alkali)	23+03EB	–	–		3.13	3.1

RenewA™ sections to minimize carryover due to traffic. The results are shown in Table 11. Cores were taken in each section near center slab and within the inside wheelpath to determine any change in the static modulus of elasticity. Some of the cores were needed to obtain compressive strength results for estimating the 40% limit used in the test so that not all samples were run. Impact-Echo wave velocities were also determined adjacent to where the cores were taken to see if this technique might provide a field method for monitoring of concrete deterioration due to ASR. All cores, dust samples and Impact-Echo readings were taken on April 23 and 24, 1996 to allow time for any effects due to the treatment to become noticeable. The overall static modulus values obtained for these cores are all significantly lower than expected. For comparison purposes the static modulus for the control section of the new construction at Chamberlain was 5.34×10^6 at 90 days. The same coarse aggregate was used in both pavements so the drop in modulus must be due to AAR, confirming the sensitivity of the static modulus to deterioration which may not be apparent with other physical tests. Interpreting the static modulus

results is difficult due to the small sample size and the variability of the condition of the concrete from sampling site to sampling site. Fortunately, because cores from before and after treatment were taken within about 6 inches of each other, the results indicate there is a small positive effect from the treatment which is statistically significant. A t-test on the cores taken from the slab centers yields a mean difference of 0.226 which is significant at the 95 % level (P - 0.039). The untreated moduli averaged 2.974×10^6 whereas treated sections averaged 3.199×10^6. The Renew™ and RenewA™ cores were combined for this analysis because of the small sample size (n=5). A similar analysis done solely on the RenewA™ center slab cores gave similar results (mean difference = 0.170; P ~ 0.040) with n=3. In both cases the statistics are robust enough to conclude there is an improvement in the static modulus. A comparison of the pretreatment static modulus values with the untreated posttreatment controls shows no significant difference indicating that any changes cannot be assigned to accelerated deterioration.

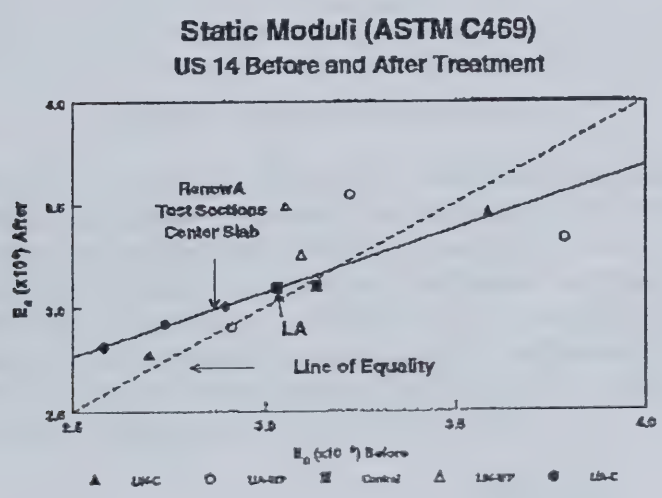

Figure 6: Static Modulus of Elasticity Results

Samples taken from the wheelpaths did not directly support this supposition. Figure 6 is a plot of the modulus values before and after treatment and may help to explain the discrepancy. Both control samples lie adjacent to the line of equality which means there was no real change in modulus during the 6 months between samples. Only two of the other points lie below the line of equality and both of these had modulus values far greater than any other cores in the pretreatment sample. In fact both of these modulus values are more than one standard deviation from the mean and the fact that the posttreatment values were much lower is probably indicative of the anomalous nature of these cores compared to the rest of the pretreatment samples. The fact that 7 out of 10 modulus values increased while one remained the same argues in favor of this interpretation.

Interestingly, the response for the RenewA™ sections was linear with roughly a 6% increase in modulus. Whether this is a fluke or a real response is anybody's guess without further testing but if it is a real effect there may not be any difference between the three sections due to increasing lithium dosage. Lithium penetration data supports this view as there was no significant depth of penetration which means any change in ASR gel expansivity caused by absorption of lithium ions occurred only in the topmost ½ inch of concrete. Additional lithium added to this section may not be of benefit until after lithium ions have had a chance to migrate deeper into the concrete.

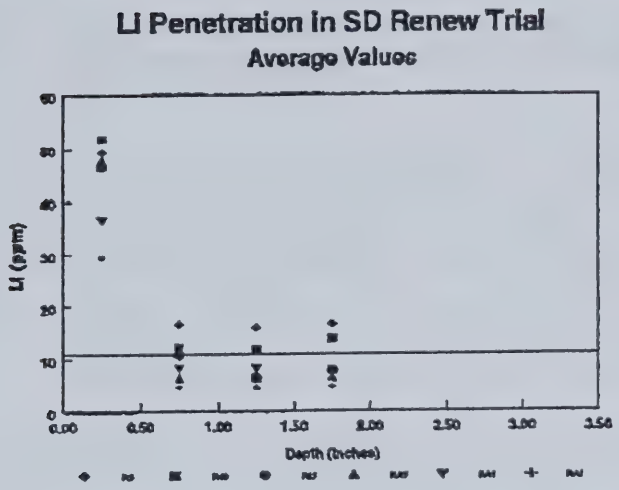

Figure 7: Lithium Penetration Results

Figure 7 illustrates the average lithium penetration profiles for each test section obtained from five randomly selected sample sites in each segment. Dust samples were taken at ½ " intervals to a depth of 2 ". The line in the figure is the average for control samples and none of the treatments exhibited Li^+ concentrations significantly greater than the control except in the topmost ½ ". Renew™ had higher concentrations of Li^+ at all application rates and appears to penetrate into the concrete deeper than RenewA™. Surprisingly, RenewA™ exhibited a linear reduction in concentration with increasing application rate which possibly indicates that the crystallization of the lithium salt between applications may actually plug the pore structure of the concrete and reduce penetration. The major visual difference between the Renew™ and the RenewA™ test sections was the wet appearance of the Renew™ sections even 24 hours after application. The RenewA™-treated concrete appeared wet for several hours after application but began to dry during the latter part of the afternoon of the application. This difference in behavior may explain why the Renew™ had somewhat higher amounts of lithium at depth.

Discussion

Preliminary indications from the test application on US 14 near Wolsey point to a slight increase in static modulus of elasticity in the concrete which substantiates the SHRP experimental application in Nevada. Further testing on this pavement is tentatively scheduled for June, 1997 and this second round of data collection should provide confirmation of the effects noted above. More than 18 months will have elapsed since treatment with Renew™ and RenewA™ and the positive benefits of these applications should increase as more lithium disperses into deeper portions of the pavement. One point needs to be made about the rate of penetration of the Li^+ ions into the concrete and it concerns the locations where samples were taken. All cores and dust samples were obtained at points on the pavement surface which exhibited no visual distress to insure that elastic modulus and penetration data would be as uniform as possible. The amount of lithium penetration and degree of possible benefit at these sampling points represent a minimum compared to those sections where ASR cracking is evident as the cracks provide channels for much more rapid ingress of solution into the concrete.

Use of LiOH for these applications was not considered advisable for two important reasons:

1) The use of a neutral salt solution which does not contain OH^- ions eliminates handling and safety problems associated with a high pH liquid rated as a hazardous material.

2) Avoiding the introduction of these same extraneous OH^- ions into a concrete containing reactive aggregates seems a prudent measure. Even if the Li ions convert existing ASR gel into a nonexpansive form the addition of OH^- into the concrete may accelerate the reaction and minimize the effectiveness of the lithium. In addition, OH^- ions are far more mobile in a concrete matrix than Li^+ ions and will infiltrate into deeper levels of the concrete where no Li^+ ions are present to mitigate the effects of the OH^- reactivity.

No concrete conclusions as to the effectiveness of either treatment or the various dosages used is possible based on these early results. Although the initial effects appear promising further monitoring will be necessary to determine the magnitude of any benefit derived from using these chemicals.

Conclusions

1) ASTM TP-14 is clearly a preferred test method for the rapid screening of aggregate for potential ASR.

2) The use of coefficient plots based on a second degree polynomial fit of TP-14 expansion data improves the interpretation of TP-14 test results, especially in regard to determining the effectiveness of mitigation techniques such as fly ash addition or use of lithium.

3) The degree of ASR mitigation achieved is a complex function of the reactivity of each aggregate as well as the amount of fly ash or lithium added to the concrete and the composition of the fly ash and cement used. Combinations must be tested individually to determine the optimum rate of addition of a given admixture.

4) Class C fly ash should be avoided in combination with a reactive aggregate unless testing indicates it is capable of reducing reactivity to acceptable levels. Typically, this may require the use of a high volume fly ash concrete with undesirable limitations.

5) Lithium hydroxide should not be used as either an admixture for new concrete or as a treatment solution for existing ASR distressed concrete. Alternative, neutral products such as LifetimeN™ and Renew™ are available, effective, safe and do not constitute a hazardous material.

6) No problems were encountered in the use of various lithium-based admixtures for new concrete construction. LifetimeN™ appears to be the admixture of choice

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16. Abstract

This research involved participation in an FHWA-sponsored round robin testing program using the AASHTO TP-14 procedure for accelerated testing of Alkali-Silica Reactive aggregates. In addition thirteen portland cement concrete pavement test sections were constructed incorporating fly ash, lithium compounds and pozzolan modified cement as mitigation techniques. These test sections will be monitored for four more years to gauge the effectiveness of the various treatments using static modulus testing and comparing all results against a control section with no treatment. Several of the treatment protocols involved the use of a combination of an F fly ash with different addition levels of lithium admixtures which constitutes the most potentially economical method for mitigation using lithium compounds. Six test sections using various amounts of neutral lithium compounds on an existing pavement suffering from severe ASR distress were also established by spraying solutions onto the slab. Initial results, based on static modulus values before and after treatment, are encouraging but further data must be collected before conclusive evidence of a positive benefit.

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due to its minimal effect on concrete properties.

7) Preliminary results from the treatment of an existing pavement with severe ASR-induced distress are favorable but more data is necessary before any conclusions can be drawn as to the effectiveness of the treatment.

Recommendations

1) The SDDOT should adopt the use of LifetimeN™ lithium-based admixture as an alternative to using a Type V/ Modified F fly ash combination to mitigate ASR reactivity in concrete where an aggressively reactive aggregate is used (TP-14 expansion > 0.25%). The LifetimeN™ would be added to a low alkali Type I-II cement/ Modified F fly ash concrete mix in an amount sufficient to suppress reactivity but not used as a sole mitigation measure to improve the economics of its use.

2) ASTM TP-14 testing should be used to determine the optimum amount of LifetimeN™ required for a given aggregate/fly ash combination. Various amounts of LifetimeN™ should be added to the mix water used in fabricating the mortar bars and the concentration of Li^+ ions in the soak solution measured to compensate for leaching of lithium during the test. The minimal amount of LifetimeN™ necessary for mitigation based on coefficient plots of the expansion data should be specified for addition to concrete.

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